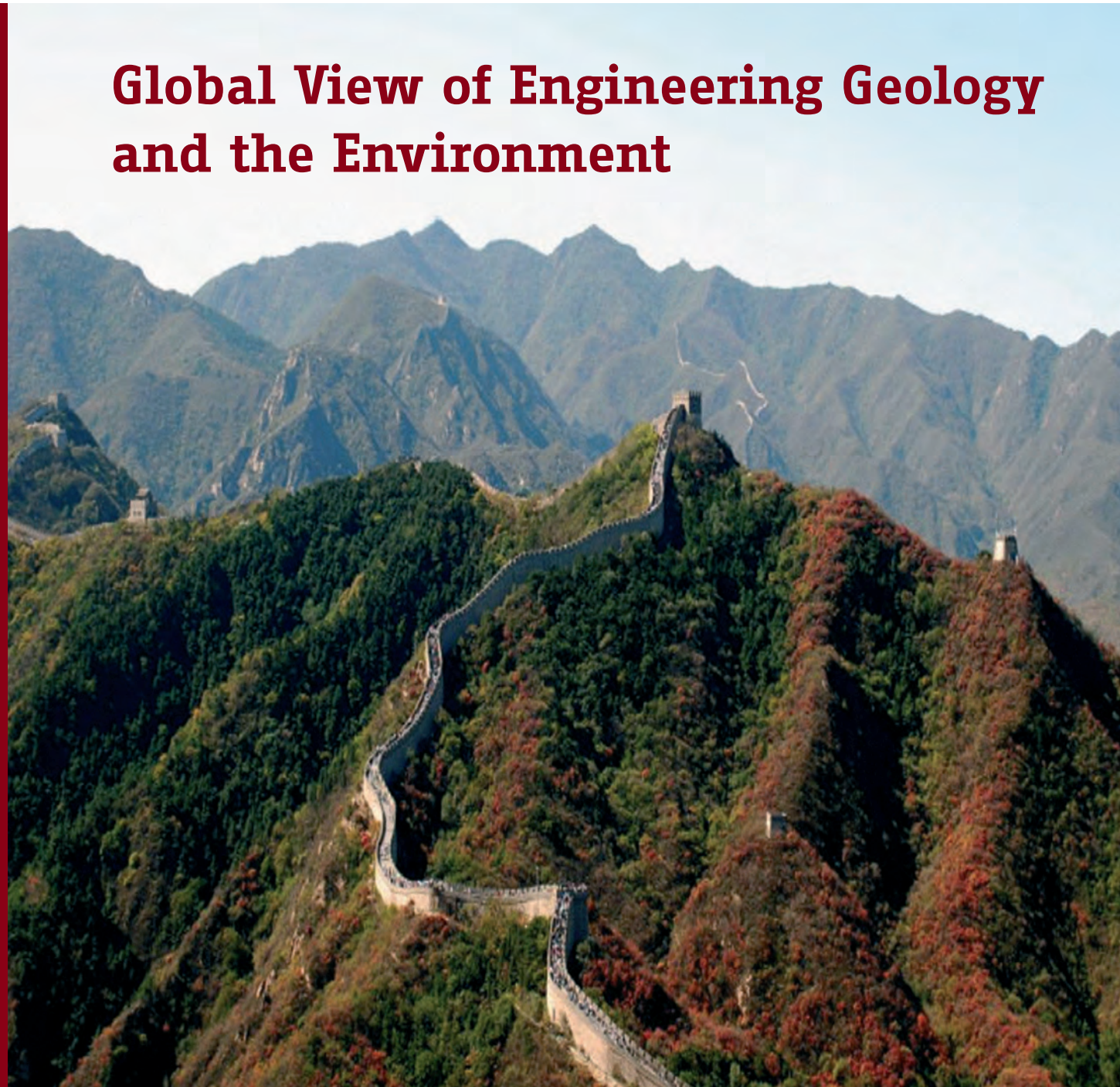


# Global View of Engineering Geology and the Environment

**Editors: Faquan Wu  
Shengwen Qi**



**help**

**open**

# Table of contents

Preface

Editorial board

Symposium credits

Keynote lectures

Theme 1: Crustal Stability and Dynamical Geo-hazards

Theme 2: Engineering Geology in Major Construction Projects

Theme 3: Urbanization and Coastal Development

Theme 4: New Ideology and Technology in Engineering Geology

Theme 5: Structure & behavior of Soil & Rock Mass

Theme 6: Geo-hazards in Karst and Loess Areas

Author index

[help](#)

[exit](#)

[main menu](#)

[search](#)

[<](#) [>](#)

# Keynote lectures

Meta-Synthesis in the engineering geology  
S. Wang

Technical criteria for the design of foundation slabs and perimetral wall in difficult terrain in South East Madrid, Spain  
C. Delgado Alonso-Martirena & F. Escolano Sánchez

Assessing the stability of a natural slope  
D.M. Cruden & C.D. Martin

Understanding Darcian flow leads to understanding aquifer mechanics  
D.C. Helm

Physico-chemical mechanics of clay soils  
V.I. Osipov

[help](#)

[exit](#)

[main menu](#)

[search](#)

[<](#) [>](#)

# Technical criteria for the design of foundation slabs and perimetral wall in difficult terrain in south east Madrid. Spain.

C. Delgado Alonso-Martirena.

*Dr. Civil Engineering. Professor Polytechnic University of Madrid. Spain.*

F. Escolano Sánchez.

*Dr. of Geology. Professor Polytechnic University of Madrid. Spain.*

**ABSTRACT:** This paper explains a procedure for the choice of ballast modules used for the design of direct continuous foundation in karst terrain. The presence of dangerous cavities is introduced in this procedure thereby evaluating risk failure. It also provides pertinent guidelines to direct the geotechnical survey of the terrain.

## 1 INTRODUCTION

For the last decade the growing extension of Madrid has led to the construction of housing and industrial estates to the east and southeast of the city centre. The terrain in these areas is chiefly gypsum rock affected by “dissolution” phenomena, covered with clayey material.

Karst cavities in the gypsum rock are an important concern for all specialists involved in the foundation design for construction projects. In this respect, slab foundations have advantages over isolated footing foundations and over deep pile foundations.

## 2 PROBLEMS AFFECTING FOUNDATION CONDITIONS.

There are three varieties of rocky layers subjected to karstification (figures 1a, b, and c):

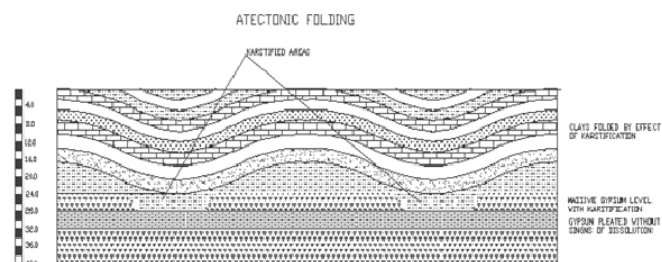


Figure 1a: Folding atectonic zones.

Folds: during their deposit part of pre-existing cavities collapse due to the geostatic weight of the

materials, giving way to the formation of folds with several dozens of metres radius in the clayey layers characteristic in the south and south-eastern areas of Madrid. The posterior processes of sedimentation and erosion conceal the presence of these folds making their identification difficult.

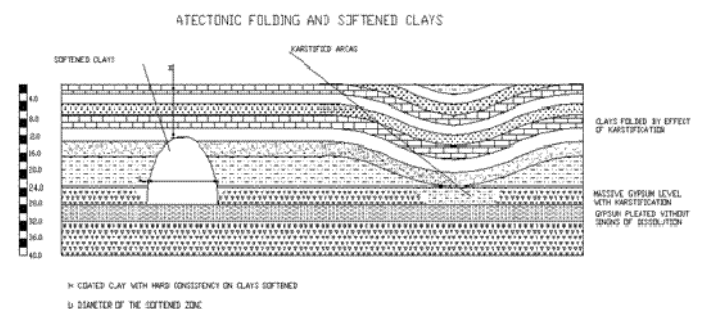


Figure 1b: Softened clay bags.

Capsules of softened clays: the collapse of cavities does not always cause folds in upper layers. Occasionally the upper layers do not break in spite of covering a cavity, and maintain horizontal layering which conceals a weak area in the contact with the rocky layer. The collapsed area is formed of low consistency clays which are not consolidated with the weight of sediments deposited later

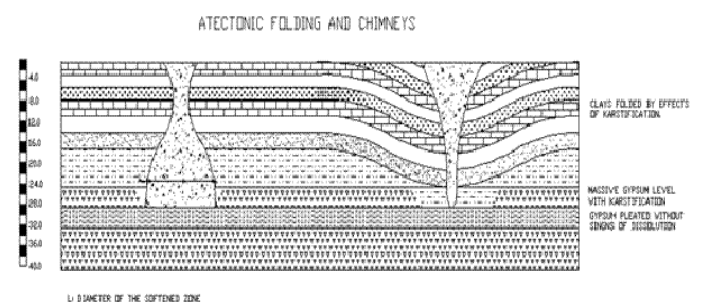


Figure 1c Chimneys.

Chimneys: both the low points in the folds as well as the softened clay capsules are points of weakness where, in a selective manner and at certain times the karstic process is reactivated creating chimneys, which eventually reach the surface. These chimneys are later filled with low consistency materials.

The diagrams in figure 1 show the different problems regularly appearing in the East Madrid area known as "Ensanche de Vallecas", where the rocky karstified layer is often found beneath the clayey covering with a depth varying between 15 and 30 m.

A geometric parameter of the softened clay capsules is also shown which is the dimension plan of the collapses (L) This parameter is the total minimum gap/space or free span, being bridged by the upper layers.

In the case of the chimneys one can talk about a diameter (D) which defines the size of the channel, in its maximum dimension.

Also, in the softened clay bags the geometrical parameter (H) is important, as it represents the minimum height of the horizontal clay layers which bridge the weakened area.

### 3 DANGER OF THE KARST PROCESS

The karst process which appears in the East and Southeast area of Madrid entails a clear danger for future urban development. In presently built-up areas, such as Santa Eugenia, there are several pathologies directly related to the subsidence phenomena.

However, the phenomenon has not been classified as catastrophic due in part to the fact that the karstic dynamic in the area is slow because:

- The climatic conditions are dry and rainfall low.
- The chalk material is of low permeability
- There is clayey material in upper layers in the Tosco-Peñuela contact

All these factors make the rainwater which infiltrates and circulates through the chalk massif not to be capable of reproducing or activating past karst dynamics. This is true only when human action does not alter the conditions of balance of the surroundings.

In this sense, there are two risk factors introduced by manmade action:

- Earth excavation which uncovers the chalk material stripping it of the protective impermeable covering.
- Drainage leaks or other water ducts placed in ground where karst dynamics have acted beforehand thus producing the washing of low compact materials which fill cavities, galleries and chimneys.

Thus, the karstic process is a danger for future urban development with two clearly differentiated as-

pects which affect different moments of the development.

On one hand, the levelling and urbanization works should take into account the possibility that these actions may reactivate the karstic dynamics and should, therefore, utilize construction procedures which guarantee the stability of the terrain during works and the life time of the urbanization.

In a second instance there is the construction of structures for residential, commercial and garage use. On this occasion the chief risk lies in the non-detection of the problems left by the karstic dynamics so that foundations are laid on material of insufficient bearing capacity.

### 4 CONTINUOUS FOUNDATIONS

The types of foundations normally used for building structures in the new areas of development in south-east Madrid are:

- Direct foundations with isolated elements such as footing or well
- Direct foundations with continuous elements such as slabs or strap footings
- Deep foundations with piles

These three types of foundations mentioned are suitable for these areas but technical justification is needed which would take into account the peculiarities mentioned above.

Below the procedure for calculation of the slabs and strap footings taking into account the risk of the karstification is explained.

#### 4.1 *Direct foundations with continuous elements.*

Foundation slabs or strap footings are structures which produce the deformation of a volume of terrain with dimensions of the same magnitude as the width of same. In the area of Ensanche de Vallecas where the position of the rocky layer can be found at depths varying from 15 to 30 metres, the layer of clays situated on the rock is within the active zone of the slab deformation.

Slab foundations have an advantage over other direct foundations which cover wider range of terrain, in such a way that certain rigidity differences are averaged out over all the active area and their deformation is more homogenous. They also have a greater capacity for bridging the karstic singularities and their softened areas.

At present the most widely used calculation procedure for structure design of continuous foundation elements is based on the coefficient of ballast method. This is based on the hypothesis that for the range of work pressure, the terrain responds with settlement directly in proportion with the pressure at



each point. The coefficient of proportionality is precisely the ballast module.

Calculation programmes normally allow for variable ballast modules from one point to another of the slab. This is precisely the manner in which, by using several ballast modules placed in different positions each one corresponding to a calculation hypothesis the presence of softened areas under the continuous foundation can be modeled.

The problem, therefore, is to determine which ballast should be used and how they should be distributed.

At present there is some controversy when choosing the ballast module which should be used in the design of continuous foundations, independently from whether they are in karst terrain or not. The criteria followed in this article consists of obtaining a ballast module from the best estimate available of the total settlement of the slab. Thus, the best ballast module is that which reproduces most faithfully in the structure the settlement obtained by geotechnical analysis.

In addition, in order to facilitate their practical use in the design of these elements, the representative ballast modules of each slab area should be constant. That is to say, the slab calculation should be carried out with two values of ballast module which would only vary by areas in the different calculation hypotheses.

The estimate of the total settlement of the slab is based on considering the natural terrain as an elasto-plastic semi-space, limited at a certain depth by a non-deformable rocky layer whose position corresponds with the beginning of the chalk layer, while the deformation area corresponds to the overlying clays.

The slab settlement estimate is carried out in the first stage by the method of finite elements which would analyse the effect of the presence of softened areas. This method is often out of reach of the foundation recommendations of a building geotechnical report, so that, based on the observations in this first model, a second model is proposed by which they are obtained more rapidly and easily with the guarantee that they are within the correct order of magnitude.

#### 4.2 Maximum settlement estimate by the finite elements method.

By using this method, different geometric situations have been modelised in which the uniform charge is situated on a clay layer which is at the same time supported by a non-deformable layer. A softened area could exist in the core of the clays.

The softened area is represented as a semi-circle of L diameter with H cover. The L and H values have been modified in the different models to obtain

a graph representing the variation of the settlements according to these two parameters.

The calculation mesh used is of similar form to that shown in the following (figure 2)

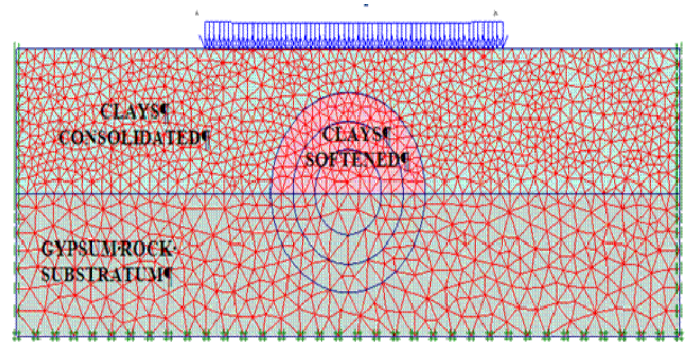


Figure 2: Modeling by finite elements of a load in a clay layer located on a non-deformable stratum. The clays may contain a softened area.

Other parameters necessary to develop the model are shown on the following table 1.

Table 1: Calculation parameters

| Lithology          | Apparent specific weight (KN/m <sup>3</sup> ) | Cohesion (KN/m <sup>2</sup> ) | Friction angle (°) | Deformation modulus (KN/m <sup>2</sup> ) | Poisson coefficient (ν) |
|--------------------|---|-------------------------------|--------------------|--|-------------------------|
| Massive gypsum     | 20,0  | 600                           | 25                 | 500.000                                  | 0,33                    |
| Consolidated clays | 18,0  | 60                            | 25                 | 50.000                                   | 0,33                    |
| Softened clays     | 18,0  | 20                            | 25                 | 500                                      | 0,33                    |
| Uniform load       | 50 KN/m <sup>2</sup>                          |                               |                    |  |                         |

The observation of the deformation of these models indicate that the slab settlement has two components. The first, due the general deformability of the clays and the second, due the presence of the softened area. The area of influence of this second settlement is over the projection of the softened area. These two deformations can be clearly seen in the following graph (figure 3).

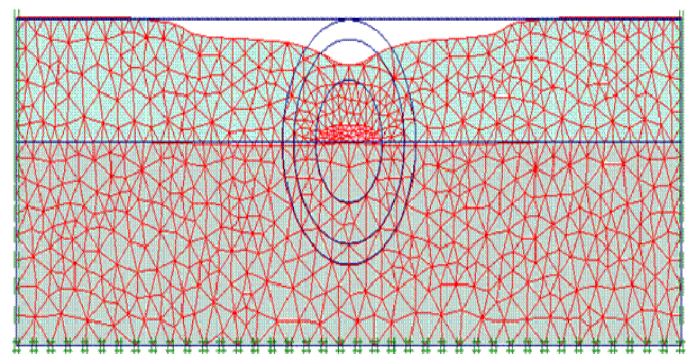


Figure 3: Deflected calculation net.

#### 4.3 Settlement calculation based on analytical formula

These previous observations indicate that the total settlement of the slab can be obtained as the sum of two components.

The separation of these two deformations permits analysis of each one in an independent manner through simple analytical formula. The two mentioned deformations are:

- The settlement due to a uniform load on the consolidated unit clays situated over a non-deformable stratum.
- The settlement due to bending and shear strain of the covering of consolidated clays which bridge the softened clays considered as a simply supported beam.

For this second deformation the settlement will be based on the thickness of the consolidated clays over the softened clays and on the diameter of these.

#### 4.4 Estimate of settlement without presence of softened areas

The determination of the settlement of a uniform charge of width B over the consolidated clays of H<sub>1</sub> power, situated over non-deformable layer is a widely studied problem.

In addition, for ranges of pressure which are normally found in the elastic area of the constitutive equations of materials and for width of slab B equal or greater than the power of compressible layer H<sub>1</sub>, the value obtained from the elastic module without lateral deformation can be considered representative. That is to say.

$$s_1 = \frac{\sigma}{E} H_1 \cdot \frac{1-\nu}{1-\nu-2\nu^2} \cdot \rho \quad (1)$$

Where: s = settlement;  $\sigma$  = Uniform load applied; E = deformation module;  $\nu$  = Poisson coefficient;  $\rho$  = Coefficient considering the lateral deformation

The  $\rho$  coefficient is adjusted from the finite element method results, so that when the diameter of the softened area is zero, the settlement of both methods coincides. The value that best adjusts to this condition is  $\rho=1.2$

Based on the value of this settlement the ballast module of calculation can be obtained for a slab resting on the consolidated clays provided no softened areas are present.

This module will be called K<sub>1</sub>. Its value is shown on the following graph (figure 4).

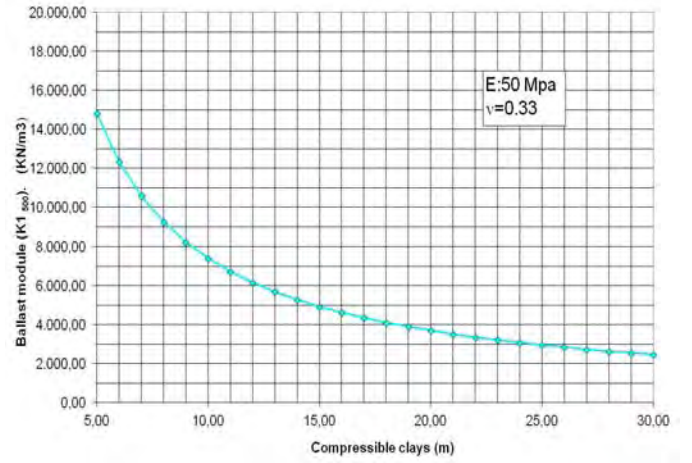


Figure 4: Ballast module for slabs on clayish soil.

As there is a lineal relationship between the deformation module and the ballast module, the value K<sub>1</sub> for any other deformation module (E) of the consolidated clay layer can be obtained by the expression.

$$K_{1(E)} = \frac{K_{1(500)}}{500} E \quad (2)$$

#### 4.5 Estimation of the settlements in presence of softened areas

The estimation of the settlement of the slab in the softened clay area is determined by considering the consolidated clays as a simply supported beam subjected to a uniform load (figure 5).

The high rim of this beam is the covering of consolidated clays over the softened area, whereas the span of the beam is in accordance with the diameter of the softened area. This calculation model is shown in the following figure 5.

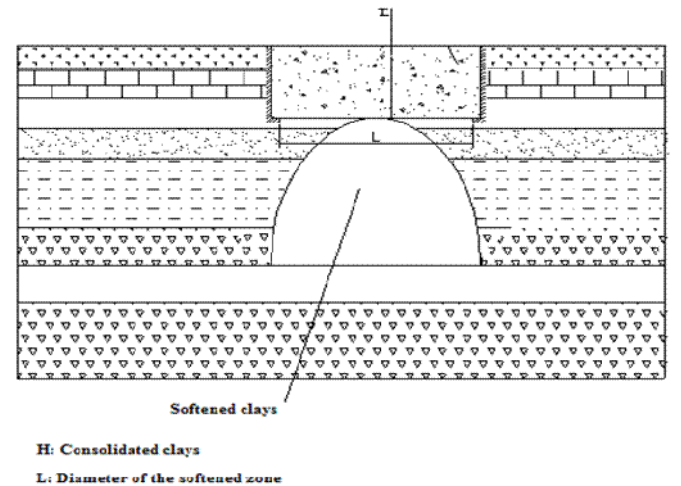


Figure 5: Modelling calculation of the deformations over cavities.

The beam has a deformation E module equal to the consolidated clay and I inertia equal to:

$$I = \frac{1}{12} H^3 \quad (3)$$

The settlement of the beam due to bending and shear deformation is equal to:

$$s_2 = \frac{\sigma L^4}{384EI} + \frac{\sigma L^2(1+\nu)}{4HE} \quad (4)$$

where  $\sigma$  represents the uniform working load.

The ballast module may be obtained from this relation, as the quotient between the working load and the total settlement.

$$K_2 = \frac{\sigma}{s} \quad (5)$$

The ballast module calculated from this expression for different values of H and L can be seen in the following (figure 6).

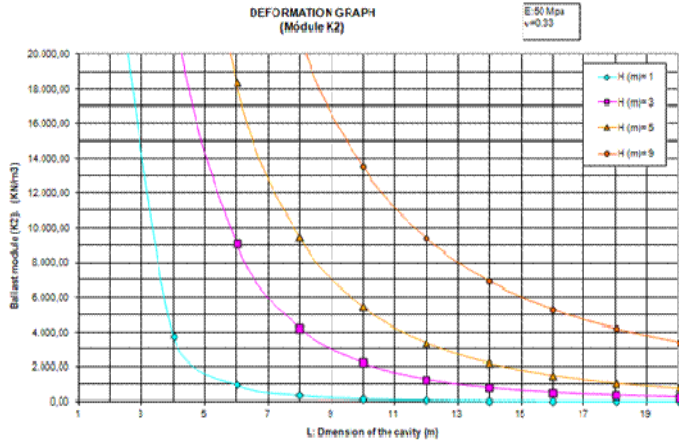


Figure 6: Modelling calculation of the deformations over cavities.

In the same way as for module K1, module K2 is proportionate to module E so that for another deformation module it is immediate.

The total settlement in the softened clays area will be same as the sum of settlement  $S_1$  and  $S_2$  and thus the ballast module in the softened clays area will be equal to:

$$Kr = \frac{\sigma}{s_1 + s_2} \quad (6)$$

Which operating the following is obtained

$$Kr = \frac{K_1 K_2}{K_1 + K_2} \quad (7)$$

Which corresponds to the addition of two serial springs.

#### 4.6 Verification of analytical method from FEM method

The validation of the analytical method is carried out by comparing its results with that of the FEM finite elements model. The graph in the following figure shows the results of the two methods in the axes of coordinates H and L previously defined (figure 7).

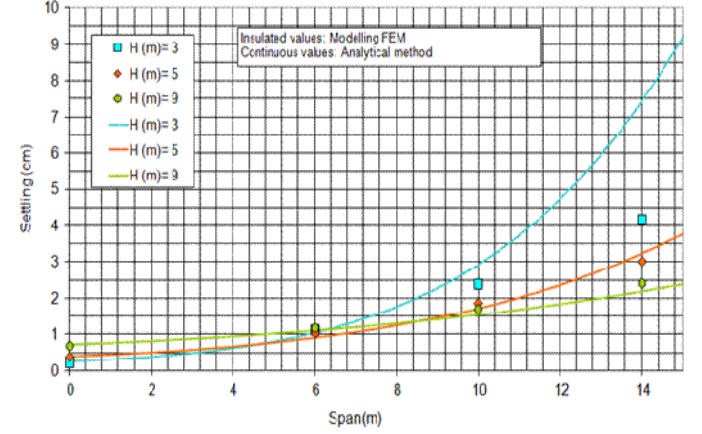


Figure 7: Comparison of the analytical method and the FEM.

The results obtained indicate a good coincidence between the two methods. In particular, the following aspects:

When there is no softened area, that is to say, for  $L=0$ , the two methods coincide practically exactly. In the two methods the same growth tendency of settlement can be observed. That is to say, for a H constant, settlements grow on increasing L. At the same time, the growth speed decreases as H increases.

There is maximum difference between the two methods when the span gap L is large in relation to the consolidated clays (H). In these situations the cavity form begins to influence the FEM method.

The estimated settlements by analytical method are equal from the point of view of practical application or higher than those calculated by the FEM method.

In view of these results it can be concluded that the analytical method described obtains, chiefly in practical situations, values similar to those of Finite Elements. Where these methods are not the same, the analytical method gives higher values of the ballast module so that it gives way to stress in the foundation elements also higher and therefore it is a method which tends to decrease the risk of fracture of the slab or strap footings

#### 4.7 Criteria to determine the distribution of the modules K1 and K2

The distribution of the modules K1 and K2 at the base of the slab will determine its rim and reinforcement.



The geotechnical survey indicates, in an approximate manner, the distribution of the softened areas; however, even though it is possible to obtain a clear idea of their existence and size, it is difficult to be certain that no other soft areas go undetected.

By increasing the number of surveys these doubts can be avoided, complemented by boreholes and penetration tests with geophysics techniques. However, on some occasions it may be preferable to increase the stress calculation and overdimension the foundation to overcome certain doubts in the survey.

For the design of foundation slabs it is necessary to determine the rim, the base reinforcement, upper and lower, and the upper and lower reinforcement of each set of pillars. The considerations for other types of continuous foundations are similar.

It is common practice for relatively symmetrical slabs with homogeneous charge distribution to determine the worst stress in one point and extend reinforcement to all the alignments of pillars.

In the present method where two ballast methods are defined, the worst stress for slab design should be calculated based on various distribution hypotheses of same. The designer should determine which are the worst positions of the combination of ballast modules for its structure.

As a guideline, the following figure (figure 8) shows a distribution of ballast modules in four calculation hypotheses. These hypotheses should be the minimum ones to be taken into account when there is no clear idea of where softened areas or cavities could appear.

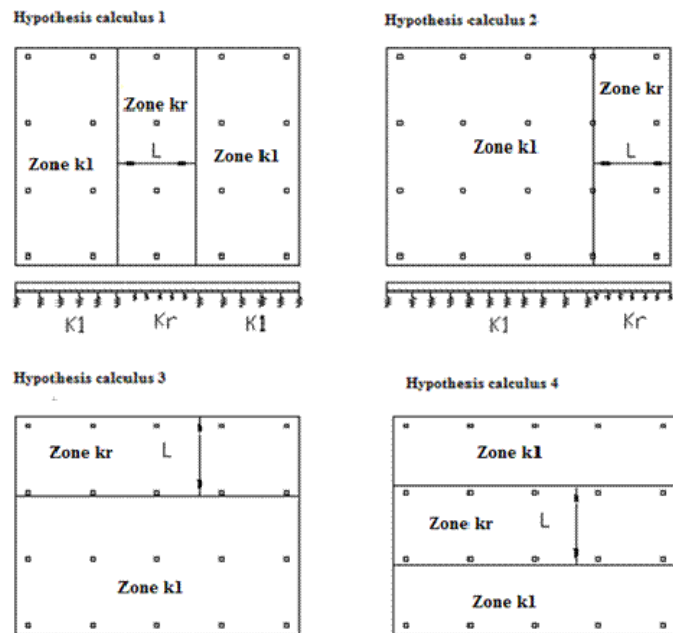


Figure 8: Distribution of ballast modules in slabs.

Additional geotechnical surveys required should be based on the study of the most sensitive areas of the deformation structure so that the new investiga-

tion can be justified with a greater dimensioning of the foundation element.

#### 4.8 Presence of karstic chimneys

Wherever karst chimney or softened areas are suspect, the capacity of the slab or footings should be evaluated.

Wherever karst chimney or softened areas are suspect, the capacity of the slab or footings should be evaluated.

In accordance with the model previously presented, chimneys are relatively cylindrical and this should be taken into account through consideration of a diameter circular. This has no support and could be present in any position relative to the slab within the “danger” zone.

The dimensions of this free diameter are determined in the ground survey by studies carried out on findings. However, based on experience on work carried out in this area in Ensanche de Vallecas these free diameter could be in the range of 2.0 to 5.0 m

#### 5 PARTICULAR INCIDENT: PERIMETRAL SUPPORT WITH DIAPHRAGM WALL.

Perimetral diaphragm walls are frequently used in the execution of basement excavation in new urban development.

The analysis of the interaction between the continuous foundation element and the diaphragm wall comes under study.

The wall is a rigid and not readily deformable element, which frequently rests on the gypsum rocky layer.

The diaphragm wall deformation is little compared to the foundation element. Under these conditions, the rim of the slab in contact with the wall is restricted in movement. This restriction could be a support or an embedment according to the design of the union.

The slab-diaphragm wall should preferably be a support so that the slab does not transmit bending to the wall. This can be done in several ways according to the following sketch (figure 9).

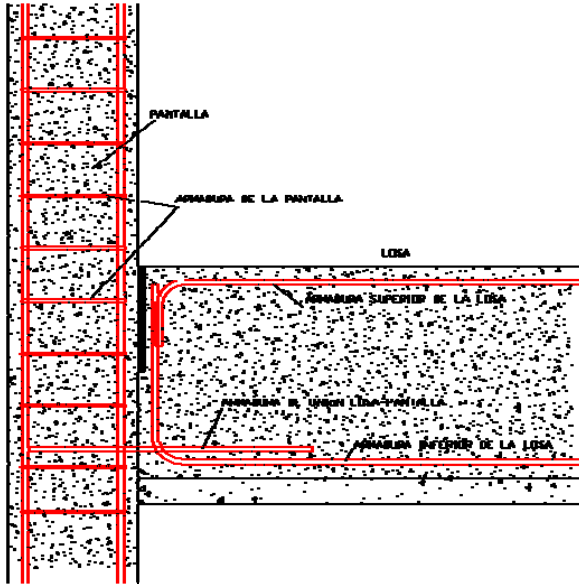


Figure 9: Support so that the slab does not transmit bending to the wall

Under these conditions the rim of the slab rests on the diaphragm wall.

So as to determine the influence of the ballast module on the stress and deformations of a slab lying on a wall, a simplified calculation model can be seen as follows (figure 10).

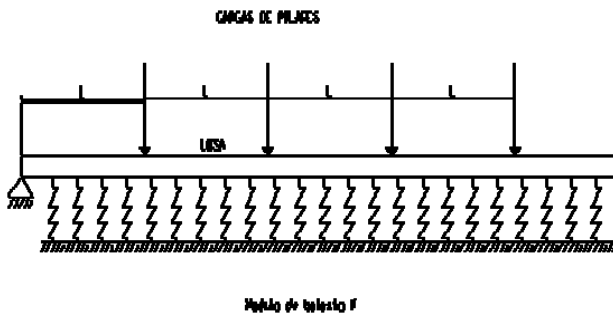


Figure 10: Pillars load.

We are talking about a semi-indefinite beam supported by a terrain with Kballast module. Vertical movement is restricted on its free border.

The beam has a constant H rim and width equal to the distance between L pillars.

A series of specific loads of equal value situated at L distance act on this beam.

The practical results of the model can be seen in the following graphs (figure 11).

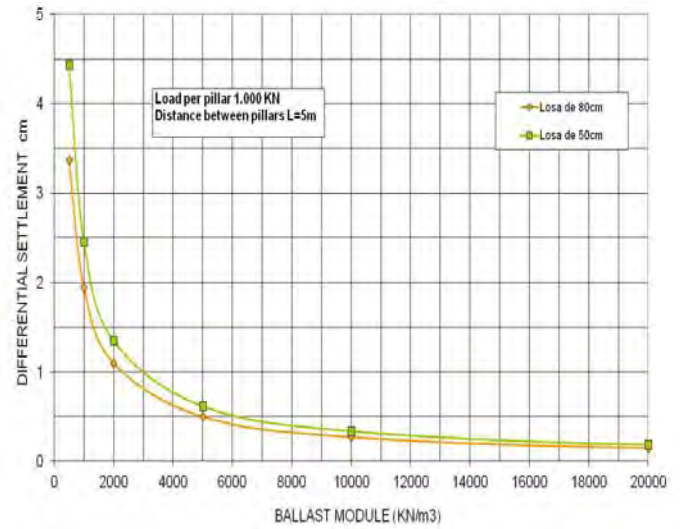


Figure 11: Graph curling of the slab.

The first one shows the deformation of the slab under the alignment of pillars close to the fixed support, thus it is a differential settlement of the arcade lying on the wall and slab at the same time. This settlement is obtained for an arcade span of 5,0m and a per pillar load of 1.000 KN.

The calculation model is linear with respect to P value and pillar load. Thus, for a load different to that indicated, the settlement would be immediately obtained by:

$$S_p = S_{1000} \frac{P}{1000} \quad (8)$$

The following points can be observed in the graph.

The differential settlement increases according to the decrease of the ballast module. However, two curves can be seen. One vertical curve corresponding to ballast modules under 2.000 kN/m<sup>3</sup> and a second curve corresponding to modules over this.

The thickness of the slab influences the differential settlement: the greater the thickness, less differential settlement. However, its influence is less decisive than the ballast module adopted.

With respect to slab stress, in the following graph we show the evolution of this stress from the positive flector moments under the first alignment of pillars according to the ballast module.

The stress evolution is obtained representing the percentage increase in the flector moment, which supposedly includes the fixed support condition on the rim of the slab, against the flector moment without this condition. Both moments are measured under the first alignment of pillars. Distance between pillars is 5.0m and load per pillar, 1.000 KN (figure 12).

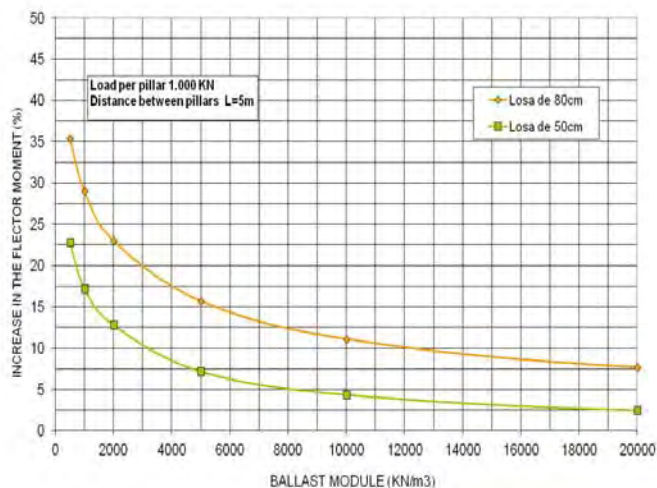


Figure 12: Graphic stress in the slab.

A tendency similar to that of deformations is observed in this graph with the following indications.

The increase of flexor moments under the pillar alignment close to the rim increases on decreasing the value of the ballast module.

The incline of this tendency becomes vertical for ballast modules less than 2.000 KN/m³.

The more rigid the slab, that is to say greater rim, the increase of flexor moments is also greater.

As a conclusion it can be said that the union between slab/diaphragm wall generates differential settlement in the structure and stress in the slab itself which should be evaluated to determine whether this foundation solution is a feasible alternative.

For ballast modules less than 2000KN/m³ and according to the load magnitudes transmitted, the presence of a perimetral wall could be inadvisable for slab foundation.

On the other hand, the slab/wall union transmits stress (vertical and bending) to the diaphragm wall which should be taken into account in its design. Bending could be eliminated with adequate union design.

## 6 CONCLUSIONS

This paper describes a design procedure for direct continuous foundation (slabs and footings) in clayey terrain with cavities and karstic phenomena situated at a certain depth.

These situations are very frequent in new urban development areas in south-east Madrid, i.e. Ensanche de Vallecas. Madrid.

The design method is based on the traditional calculation method of these elements with the ballast module. The slab rests on two different ballast modules whose position depends on the configuration of cavities detected during the terrain survey. Due to uncertainty about the position of karst areas, several

calculation hypotheses are used about the distribution of these modules. This depends basically on the position of the rocky layer under the slab, the diameter of the cavities and the covering by competent clays.

Finally, the interaction of slab/wall is frequently studied in the case of plots with perimetral walls used in the execution of foundations.

## 7 REFERENCES

- Arche, A, 1992. Sedimentology and tectonic relations, in Sedimentology Vol ICSIC. 433-489.
- Barretino raile, D., 1994. The Karst. Geoenvironmental Engineering Serie. Ed IGME. Madrid.
- Brandis, D., Del Rio, I, 2009. Last Urban Developments in Madrid Surroundings. VIII Symposium and Field Excursion Urban Geography.
- Bueno Aguado, M., Escolano Sánchez, F, 2006. Geotechnical Aspects of the new urban development's southeast of Madrid: The Vallecas PAU" Intevía paper.
- Cañaveras, J.C., Sánchez Moral, S, 2003. Paleokarstic profiles in middle Miocene Unit of the Madrid Basin. Geologic studies. 59, 53-66.
- Escario, V, 1985. Geotechnical soils synthesis of Madrid and its district. General direction of Transportation Infraestructure.
- Escolano Sánchez, F, 2005: Geological-geotechnical characterization of evaporite facies transition and Madrid Basin. UPM Digital Archive.
- Escolano Sánchez, F., Bueno Aguado, M, 2007. Special Foundation on karstic site locates at the southeast of Madrid. 14th European conference on soil mechanics and geotechnical engineering (proceedings). Vol.4, 2179-2182.
- Escolano Sánchez, F., Bueno Aguado, M, 2009. Risk analysis karst in Madrid. *Ingeopres.* 180, 32-38.
- Gonzalez Tejada, I. et al, 2009. Geotechnical characteristics of the main gypsum materials for use in embankments. *Ingeopres.*186, 24-33.
- Madrid City Council 2011. Monitoring and management of new urban developments.
- Mancebo Piqueras, J.A., Sanz Pérez, I., Menéndez Pidal, I. 2012. Water seepage beneath dams on soluble evaporite deposits: A laboratory and fiel study (Caspe Dam, Spain). *Bulletin of engineering Geology and the Environment.* 71, 201-213.
- Oteo Mazo, C., Rodríguez Ortiz, J.Mª, 2003. Systems design and geometrical parameters in the expansion of the Madrid Metro. *Obras Públicas Magazine.* 3429, 49-67.
- Parise, M.; Waele, I.D., Gutierrez, F. 2008. Engineering and environmental problems in karst. An Introduction. 99, 91-94.
- Perles Roselló, MJ., Cantarero Prados, F, 2010. Problems and challenges of multiple risk analysis planning: methodological proposals for the development of multi-hazard mapping. *Bulletin of the Association of Spanish Geographers.* 52, 245-271.
- Pueyo-Anchuela, O., Casas-Sainz, AM., Pocoví J., Ansón-López. 2011. Multidisciplinary approach for urban planing in alluvial karstic zones: Case Study from the Central Ebro Basin (Spain). *Engineering Geology* 122. 222-238.
- Rodríguez Ortiz, J.Mª, 2000. Geotechnical properties of soils in Madrid. *Obras Públicas Magazine.* 3405, 59-83.
- Romanov, D., Kaufmann, G., Hiller, T. 2012 Karstificationof an aquifer along the Birs river, Switerland. *Engineerin Geology.* 150, 9-23.

- Romanov, D., Kaufmann, G., Hiller, T., Epting, J., Hoggernbergr, P. 2010. Karstification of aquifers interspersed with non-soluble rocks: From basic principles towards case studies. *Engineering Geology* 116. 261-273.
- Tsesarsky, M. 2012. Deformation mechanisms and stability analysis of undermined sedimentary rocks in the shallow subsurface. *Engineering Geology*, 133-134, 16-29.
- Vera, J.A., 2004. Geology of Spain. Geological Society of Spain. 556-560